SWENSON SAY FAGÉT
A STRUCTURAL ENGINEERING CORPORATION

Federal Reserve Bank Building
Seismic Study for Reuse Feasibility
March 5, 2014

This report outlines our Feasibility Study findings for the existing Federal Reserve Bank building located at 1015 2nd Avenue in Seattle, Washington. The existing Seattle Federal Reserve Bank building is a 6-story concrete and steel framed building. The Federal Reserve Bank operations relocated to Renton, Washington in 2008 and the building has been vacant since then. The building currently is surplus federal property and under the McKinney-Vento Homeless Assistance Act, is being made available to qualifying non-profit organizations for reuse. The current project consists of developing a feasibility plan including rehabilitation shell and core budget for the building including necessary seismic improvements.

The structural scope of work for the feasibility study included

- Site visit(s) to make observations of the building and current building conditions.
- Review and analysis of available structural drawings for the original building and building modifications made since the original construction.
- Review of previous seismic evaluations and condition assessments for the building
- Limited seismic analysis of the building to determine potential building structural deficiencies

Drawings reviewed for this feasibility study included the following:
1949 Narramore Bain Brady & Johanson (NBBJ) Architects Structural Drawings S1-S12 and supplemental Revisions sheets 1 and 2

1996 NBBJ Seismic Retrofit and Third Floor Renovations including Structural Drawings sheets S1.1-S10.1.

In addition, we reviewed the 2002 Otak, Inc. condition assessment report commissioned by the Federal Reserve System.

Building Seismic Structural System review

Swenson Say Fagét made a brief walk-through of the existing building in late February 2104. Only limited observations of existing structure at the basement and ground floor could be made due to finishes or fireproofing placed over the structural framework of the building. Structural connection details could not be observed or verified. We reviewed the original 1949 building construction drawings and the 1996 Seismic retrofit drawings to understand the building framing and building lateral load resisting systems. The building structure consists primarily of steel framing with concrete-topped metal deck roof and floors. Limited concrete walls are present for wind and seismic lateral load resistance. The basic building configuration is two stories below grade and 4 stories above grade however the building footprint steps out on all sides at first floor such that the building wall lines for the upper four levels are discontinuous in the ground floor and basement levels. This arrangement has created a vertical discontinuity in the concrete shear wall lateral force resisting systems for the building. Horizontal inertial forces generated in the shear walls in the upper four stories do not have a direct path to foundations below and rely on lateral force transfer through the first floor diaphragm to the offset wall lines at the lower two levels. Based on our review of the original construction drawings, it appears that the first floor diaphragm was not strengthened or detailed to adequately transfer the lateral loads. In addition, the original structure supporting the upper level concrete shear walls appeared to be inadequate to support vertical shear wall overturning forces.

It appears that a previous seismic evaluation of the building was done in the mid-1990s although no formal written seismic evaluation documents were found as part of our document review. The evaluation apparently determined that the existing, original concrete walls for the upper four floors of the building were inadequate to resist seismic forces on the building. As a result, the 1996 NBBJ Seismic retrofit added an interior concrete shear wall facing to existing 8' concrete walls in the building at the upper four floors. However, the vertical discontinuity of the shear walls was not adequately addressed and an adequate shear load path between the upper four floors and the offset ground and basement walls was not created as part of the seismic retrofit work. While some structure supporting shear walls was strengthened and retrofit, in some locations the existing steel structure at the first floor supporting the shear wall above was not adequately strengthened to resist shear wall overturning effects. Boundary elements at shear wall ends resisting uplift forces were also apparently not addressed inadequately in some locations.

As part of the 2002 Otak, Inc. condition assessment report, an ASCE 31 Tier 1 Seismic Evaluation of the building was done. The results of the evaluation noted the vertical discontinuities mentioned above as well as the related deficiencies in members supporting shear walls, boundary elements and overturning effects. The report recommended further study of the issues in order to determine mitigation measures however no additional seismic retrofit work was done as a result of the study.

Seismic Study for Current Feasibility Study

For the current feasibility study, Swenson Say Faget performed a limited seismic analysis of the building using ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*. ASCE 7-10 is the basis for the current building code. Seismic forces on the building were calculated based on 2008 USGS ground motion hazard data which is the most current seismic data used for both ASCE 7-10 and the 2012 International Building Code.

As part of our analysis, the building's concrete shear wall system was analyzed taking into consideration the seismic retrofit work done in 1996. Building structure supporting shear walls was also analyzed.

Our limited seismic analysis found that the seismic retrofit work done in 1996 adequately reinforced the concrete shear walls in the upper four story structure for current building code level horizontal seismic shear forces. However we found deficiencies in the global lateral load resisting system including:

- Vertical discontinuity in the load path of the lateral force resisting system at the first floor wall offset. Inadequate first floor diaphragm strength limits horizontal shear transfer from upper floor wall lines to ground floor and basement wall line.

- Deficiencies in shear wall boundary elements and inadequate supporting structure for shear wall overturning effects. Existing steel reinforcing at concrete shear wall ends is inadequate to resist tension loads due to shear wall overturning effect. Under earthquake ground motions, existing steel would yield inelastically or fail leading to excessive building deflections and possible severe building damage or partial collapse.
- Inadequate attachment of floor diaphragms to steel members at added concrete shear walls. The concrete shear wall facing added in the 1996 retrofit dowelled the facing into the existing concrete topped metal deck floor assembly. However the original concrete topped metal deck is likely minimally attached to the steel framing supporting it. The steel framing acts as a drag element to deliver diaphragm forces to the shear walls and requires proper deck attachment to function properly.

We also found deficiencies in some non-structural components of the building including inadequate out-of-plane support for hollow clay tile and unreinforced masonry (URM) walls at the alley-side mechanical penthouse walls, interior partition walls and stair walls. Wall height-to-thickness ratios make them vulnerable to out-of-plane partial collapse under seismic loads. Partially collapsed walls could injure building occupants and block safe egress from the building.

Remediation Recommendations

Based on our limited seismic analysis of the building and the identified deficiencies, we recommend the following mitigation measures:

Vertical Discontinuity in the building Lateral Load system at first floor

- Install braced frames in the ground and basement levels below existing concrete shear walls to eliminate the discontinuity in the vertical lateral load system. See plan marks for locations and member sized. Braced frames would consist of wide flange steel columns and beams and round HSS cross bracing. The braced frames would be attached to existing steel and concrete structure using high-strength bolts at existing steel framing and drilled epoxy or expansion bolts at existing concrete. In each braced frame location, the frame in the ground level would align with the shear wall above and with the braced frame at the basement level and be attached through existing floor structures with high strength threaded rods or steel plates.

  Due to high uplift forces at the ends of the added braced frames, drilled micropiles would be installed to resist frame uplift forces. A reinforced concrete pile cap would be installed at the braced frame ends to transfer uplift forces from the frame members to the micropiles and to transfer frame compression loads to existing spread footings.

Deficiencies in shear wall boundary elements

- Install steel channels to the face of existing concrete shear walls at ends to provide adequate resistance to tension forces generated by shear wall overturning effects. See plan marks for locations. The channels would be bolted to exiting concrete using drilled epoxy bolts or expansion anchors. Where the channel reinforcing extends between floor levels, steel plates and high strength threaded rods would be used to connect the channels above and below the floor structure.

Inadequate attachment of floor diaphragms to steel members at added concrete shear walls

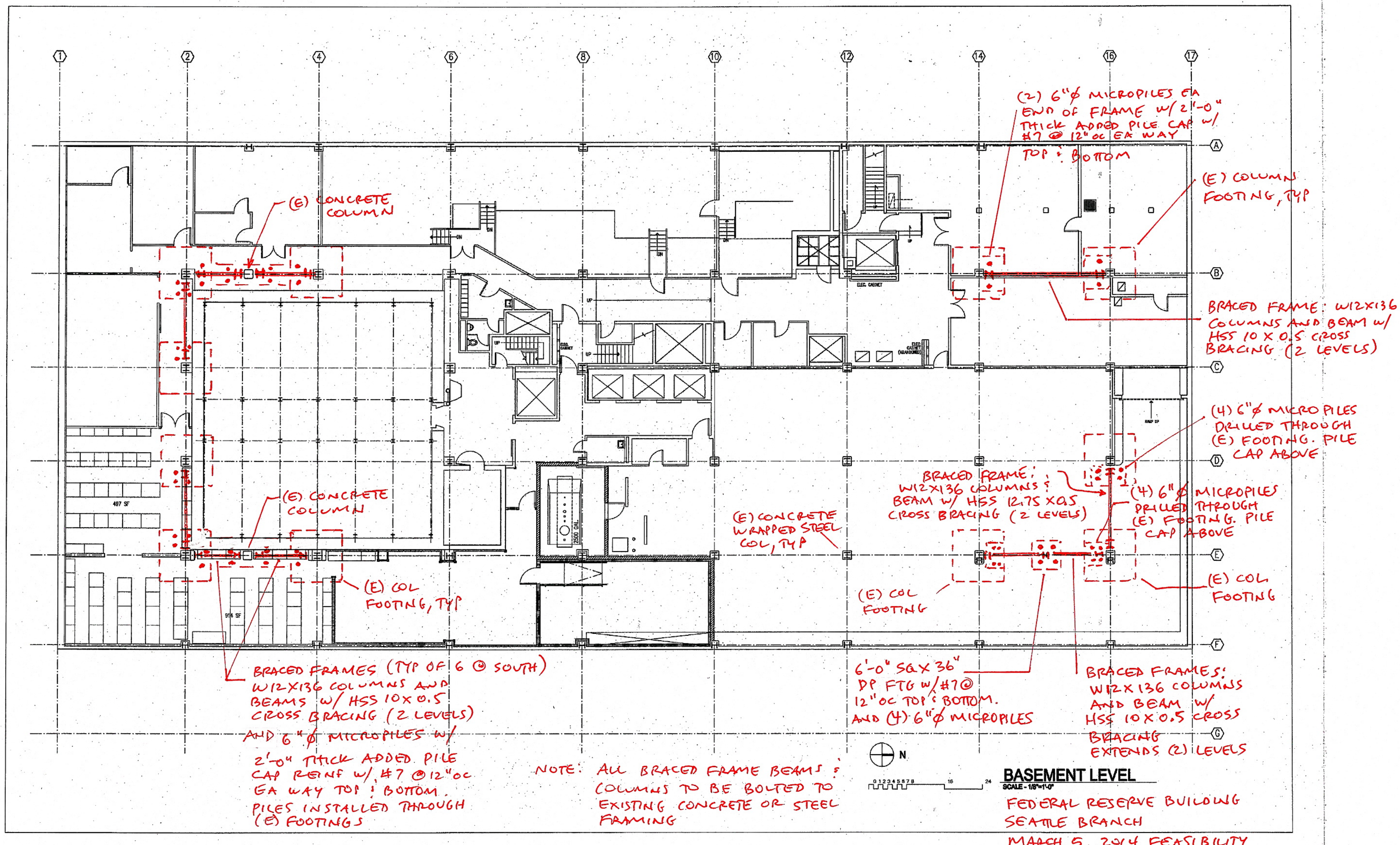
- Attach steel framing below shear walls to existing concrete topped metal deck. Deck can be attached to supporting steel with drilled expansion bolts installed from below through the top flange of the existing steel beams. Bolts would be installed at approximately 18" o.c. into the concrete filled flutes of the existing floor deck.

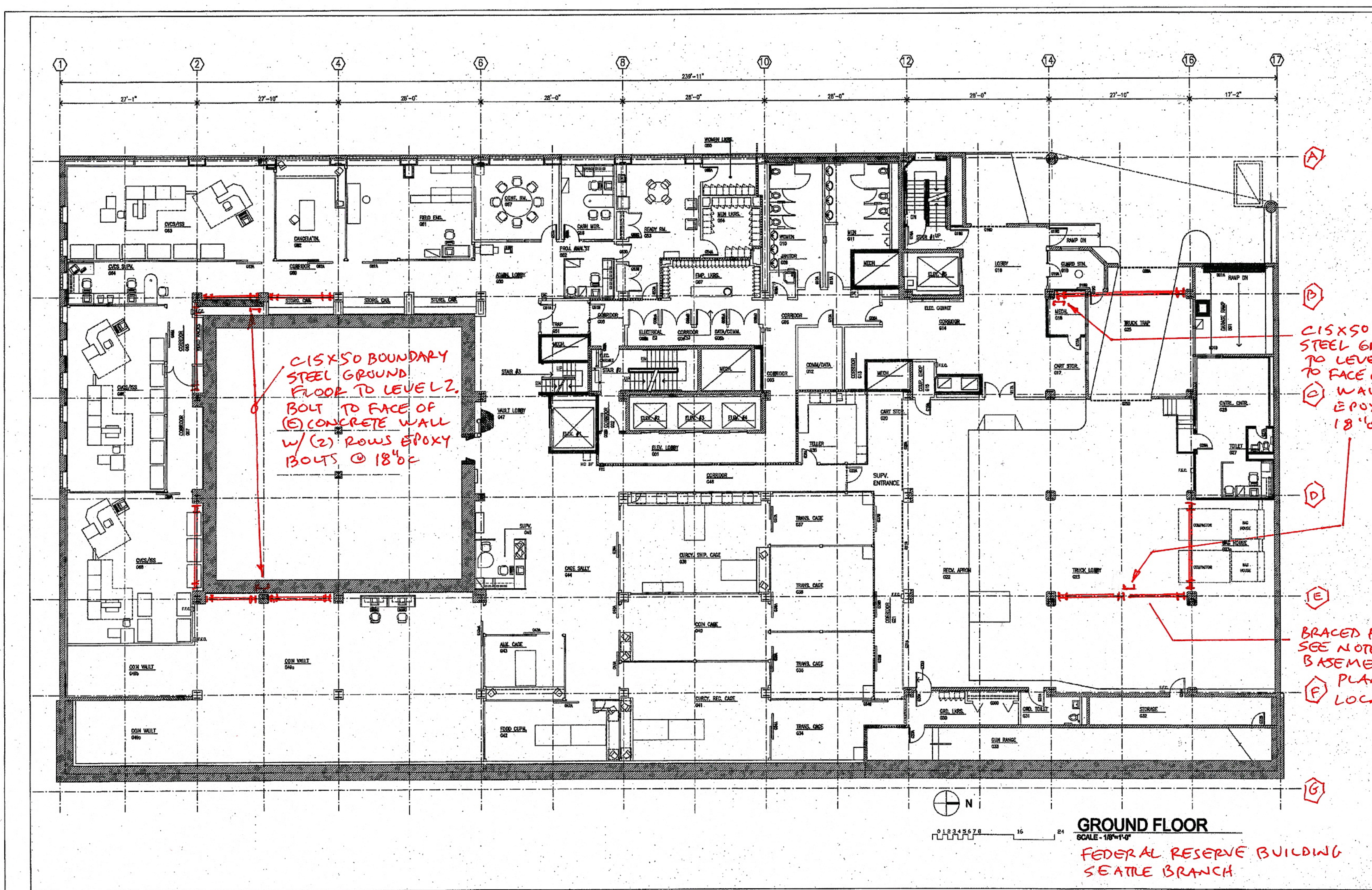
Unbraced hollow clay tile and unreinforced masonry walls

- Provide steel strong-backing at clay time and URM walls. Strong backing would consist of 6" 18 gauge steel studs and blocking attached to the face of the clay tile or masonry walls. Stud framing and blocking would be attached with 3/8" diameter drilled epoxy bolts or expansion anchors. Strong-back wall top and bottom tracks would also be attached to existing structure using epoxy bolts or expansion bolts.

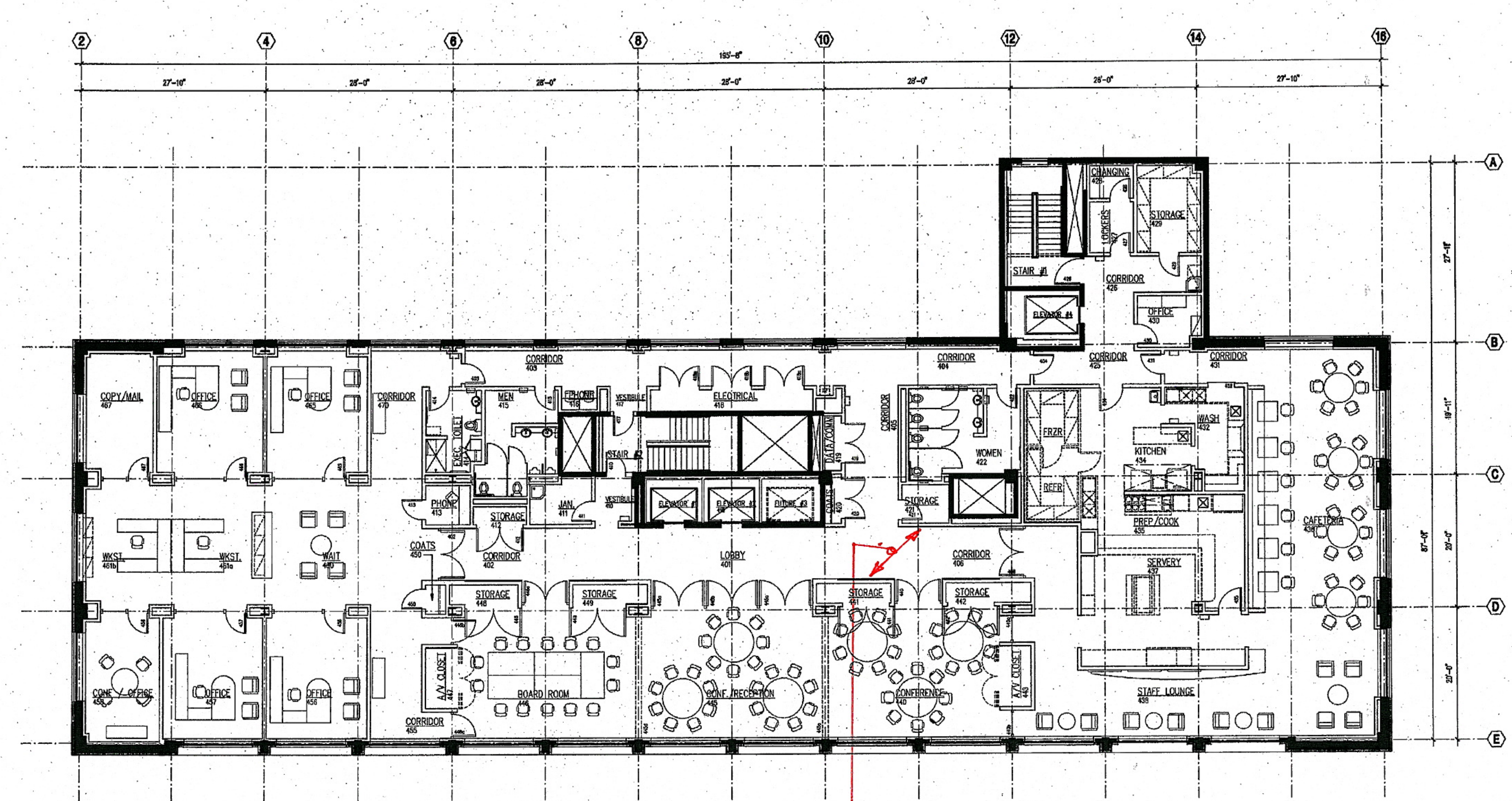
<u>Conclusions</u>

Swenson Say Faget performed a limited seismic analysis of the existing Federal Reserve Bank building and found significant structural deficiencies in the buildings lateral load resisting systems. We have provided schematic level mitigation measures for the identified structural deficiencies. Because our seismic analysis was limited in scope, there may be other building seismic deficiencies. As part of a project to undertake the recommended seismic improvements to the building, we recommend that a more detailed seismic analysis and building investigation be done to more fully identify the building's deficiencies. In addition, the 2002 Otak, Inc. building assessment noted other potential building non-structural hazards such as inadequate ceiling and light fixture bracing, insufficient exterior cladding and glazing attachment, and insufficient mechanical and electrical equipment anchorage and bracing. Swenson Say Fagét did not investigate or evaluate these other potential hazards. However, the noted hazards are typical of buildings of this age and construction type. We recommend that these potential hazards we investigated further and remediated as part of the building seismic improvements.


1
2
4
6
8
10
12
14
16
17
A
B
C
D
E
F
G
(2) 6"ø MICROPILES EA END OF FRAME w/ 2'-0" THICK ADDED PILE CAP w/ #7 @ 12"oc EA WAY TOP : BOTTOM
(E) COLUMN FOOTING, TYP
(E) CONCRETE COLUMN
BRACED FRAME: W12X136 COLUMNS AND BEAM w/ HSS 10 X 0.5 CROSS BRACING (2 LEVELS)
(4) 6"ø MICROPILES DRILLED THROUGH (E) FOOTING. PILE CAP ABOVE
BRACED FRAME: W12X136 COLUMNS : BEAM w/ HSS 12.75 X 0.5 CROSS BRACING (2 LEVELS)
(4) 6"ø MICROPILES DRILLED THROUGH (E) FOOTING. PILE CAP ABOVE
(E) CONCRETE COLUMN
(E) CONCRETE WRAPPED STEEL COL, TYP
(E) COL FOOTING, TYP
(E) COL FOOTING
(E) COL FOOTING
497 SF
914 SF
2500 GAL
BRACED FRAMES (TYP OF 6 @ SOUTH) W12X136 COLUMNS AND BEAMS w/ HSS 10 x 0.5 CROSS BRACING (2 LEVELS) AND 6"ø MICROPILES w/ 2'-0" THICK ADDED PILE CAP REINF w/ #7 @ 12"oc EA WAY TOP ! BOTTOM. PILES INSTALLED THROUGH (E) FOOTINGS
6'-0" SQ X 36" DP FTG w/ #7 @ 12"oc TOP : BOTTOM. AND (4) 6"ø MICROPILES
BRACED FRAMES: W12X136 COLUMNS AND BEAM w/ HSS 10 X 0.5 CROSS BRACING EXTENDS (2) LEVELS
NOTE: ALL BRACED FRAME BEAMS : COLUMNS TO BE BOLTED TO EXISTING CONCRETE OR STEEL FRAMING
N
0 1 2 3 4 5 6 7 8 16 24
BASEMENT LEVEL
SCALE - 1/8"=1'-0"
FEDERAL RESERVE BUILDING
SEATTLE BRANCH
MARCH 5, 2014 FEASIBILITY



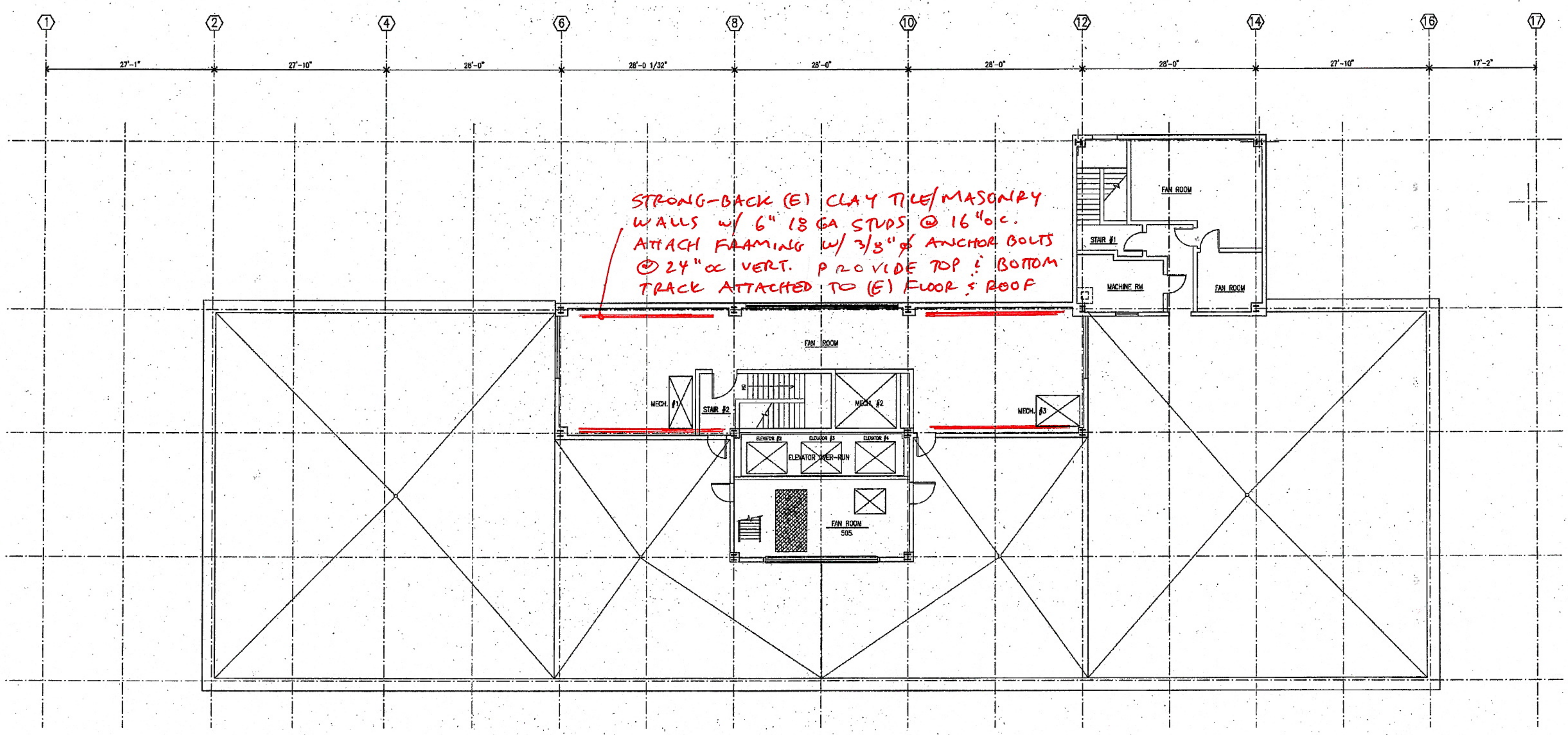
MARCH 5, 2014 FEASIBILITY



VERIFY EXISTING WALL CONSTRUCTION AT CORRIDORS, ELEVATOR LOBBY AND OFFICE PARTITIONS.
PROVIDE STEEL STUD STRONG BACKING OR DEMOLISH AND REPLACE WITH STEEL STUD WALLS IF EXISTING WALLS ARE HOLLOW CLAY TILE OR UNREINFORCED MASONRY.
TYPICAL AT LEVELS 2-4.

4TH FLOOR
SCALE - 1/8"=1'-0"

FEDERAL RESERVE BUILDING
SEATTLE BRANCH
MARCH 5, 2014 FEASIBILITY



N ROOF LEVEL

FEDERAL RESERVE BUILDING
SEATTLE BRANCH

MARCH 5, 2014 FEASIBILITY